STRUCTURES DESIGN GUIDELINES
FOR
LOAD AND RESISTANCE
FACTOR DESIGN

STRUCTURES DESIGN OFFICE
TALLAHASSEE, FLORIDA
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INTRODUCTION

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STRUCTURES DESIGN GUIDELINES

I.1 Purpose

The ***Structures Design Guidelines (SDG)***, sets forth the basic Florida Department of Transportation (FDOT) design criteria that are exceptions to those included in the ***AASHTO/LRFD Bridge Design Specifications*** published in Customary U.S. Units by the American Association of State Highway and Transportation Officials (AASHTO) for Load and Resistance Factor Design (LRFD), and hereinafter simply referred to as "***LRFD***." These exceptions may be in the form of deletions from, additions to, or modifications of the ***LRFD*** specifications. ***AASHTO/LRFD*** and the ***Structures Design Guidelines (SDG)*** must always be used jointly and in concert with the FDOT's ***Plans Preparation Manual (PPM)*** in order to properly prepare the contract plans and specifications for structural elements and/or systems that are included as part of the construction work in FDOT projects. Such elements and/or systems include, but are not necessarily limited to, bridges, overhead sign structures, earth retaining structures, and miscellaneous roadway appurtenances.

I.2 Authority

Section 334.044(2), Florida Statutes.

I.3 Scope

The use of this manual is required of anyone performing structural design or analysis for the Florida Department of Transportation.

I.4 General

This manual consists of text, figures, charts, graphs, and tables as necessary to provide engineering standards, criteria, and guidelines to be used in developing and designing structures for which the Structures Design Office (SDO) and District Structures Design Offices (DSDO) have overall responsibility. The manual is written in the same general format and specification-type language as ***LRFD*** and includes article-by-article commentary when appropriate. In addition to its companion documents referenced in Article I.1 above, the manual is intended to complement and be used in conjunction with the SDO's ***Detailing Manual***, ***Standard Drawings***, ***CADD User's Manual***, and the ***Standard Specifications for Road and Bridge Construction***.

I.5 Referenced Documents

Other FDOT Manuals referenced in this document include:

Plans Preparation Manual, Volume 1 (Topic No.: 625-000-007)
Plans Preparation Manual, Volume 2 (Topic No.: 625-000-008)
Detailing Manual (Topic No.: 625-020-200)
Drainage Manual (Topic No.: 625-040-001)
Standard Drawings (Topic No.: 625-020-300)
CADD User's Manual

I.6 Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
DSDE	District Structures Design Engineer
DSDO	District Structures Design Office
FHWA	Federal Highway Administration
FDOT	Florida Department of Transportation
LRFD	Load and Resistance Factor Design
PPM	Plans Preparation Manual
SDG	Structure Design Guidelines
SDO	Structures Design Office
SSDE	State Structures Design Engineer
TAG	Technical Advisory Group
TDB	Temporary Design Bulletin

I.7 Distribution

This ***SDG*** is furnished via the SDO web page at no charge. It is the responsibility of the designer to regularly check for additions, modifications and bulletins. Questions regarding this manual and any modifications may be obtained from the following office and address:

Structures Design Office
Mail Station 33
605 Suwannee Street
Tallahassee, Florida 32399-0450
Tel.: (850) 414-4255
http://www11.myflorida.com/structures

I.8 Registration

This manual no longer contains a registration page. It is incumbent upon the user to keep abreast of changes via the SDO web page.

I.9 Administrative Management

Administrative Management of the ***Structures Design Guidelines*** (***SDG)*** is a cooperative effort of SDO staff and the nine voting members of the Technical Advisory Group (TAG).

I.9.1. The Technical Advisory Group (TAG)

The TAG provides overall guidance and direction for the ***SDG*** and has the final word on all proposed modifications. The TAG comprises the State Structures Design Engineer (SSDE) and the eight District Structures Design Engineers (DSDE). In matters of technical direction or administrative policy, when unanimity cannot be obtained, each DSDE has one vote, the SSDE has two votes, and the majority rules.

I.9.2. SDO Staff

SDO Staff comprises the Assistant State Structures Design Engineers and Senior Structures Design Engineers selected by the SSDE.

I.10 Modifications And Improvements

All manual users are encouraged to suggest modifications and improvements to the ***SDG***. Modifications to the manual are the direct result of changes in FDOT specifications, FDOT organization, Federal Highway Administration (FHWA) regulations, and AASHTO requirements; or occur from recent experience gained during construction, through maintenance, and as a result of research. Users have suggested many other improvements to the manual. This has been particularly true with suggestions to improve the clarity of the text and to include design criteria not previously addressed by the manual or by any accompanying code or specification. Transmit, suggestions in writing to the State Structures design Engineer, 605 Suwannee St, (ms 33), Tallahassee, FL 32399

I.11 Adoption of Revisions

Revisions to the ***SDG*** are issued by the SDO as Temporary Design Bulletins or Permanent Revisions according to a formal adoption process. Temporary Design Bulletins provide the SDO with the flexibility to quickly address any modification considered by the SDO to be essential to production or structural integrity issues.

I.11.1. Temporary Design Bulletins

A ***Temporary Design Bulletin (TDB)*** is a revision to the ***SDG*** that is deemed by the SSDE to be mandatory and in need of immediate implementation. ***TDB's*** may address issues in plans production, safety, structural design methodology, critical code changes, or new specification requirements.

TDB's supercede the requirements of the current version of the ***SDG*** and may be issued at any time. ***TDB's*** are not official or effective until signed by the State Structures Design Engineer.

TDB's are effective for up to 360 calendar days unless superceded by subsequent ***TDB's*** or Permanent Revisions to the ***SDG***. ***TDB's*** automatically become proposed Permanent Revisions unless withdrawn from consideration by the SSDE.

TDB's indicate their effective date of issuance, include the reference Topic Number, are numbered sequentially with reference to both the ***SDG*** version number and year of issuance. For example, Temporary Design Bulletin No. C98-2 would be the second Bulletin issued in 1998 for ***SDG.***

I.11.2. Permanent Revisions

Permanent Revisions to the manual are made semi-annually or "as-needed." If the SDO considers an individual revision, or an accumulation of revisions, to be substantive, the manual will be completely reprinted. The following steps are required for adoption of a revision to the manual.

A. Revision Assessment
   SDO Staff will assess proposed revisions and develop the initial draft of all proposed ***SDG*** modifications for the SSDE's approval.

B. Revision Research and Proposal
   The SDO Staff will conduct the necessary research, coordinate the proposed modification with all other affected offices and, if the proposed modification is deemed appropriate, prepare a complete, written modification with any needed commentary. The SSDE's approval signifies the SDO's position on the proposed modifications.

C. Revision Distribution To TAG and Other Review Offices
   Proposed modifications will be mailed to TAG members and others allowing for no less than two weeks review time before the next scheduled TAG meeting. Other parties include, but are not limited to: State Construction Office, State Maintenance Office, State Materials Office, State Roadway Design Office, Organization, Forms and Procedures, and FHWA.

   DSDE members of TAG will coordinate the proposed modifications with all other appropriate offices at the district level.

D. Revision Review by TAG and SDO
   Each TAG member will review proposed modifications prior to the meeting where they will be brought forward for discussion The SDO will review all modification comments received from other FDOT/FHWA offices in preparation for presentation at the meeting.

   Additional review comments received by the SDO and/or DSDO's during the review process will be presented for discussion and resolution.

E. TAG Adoption Recommendation

Immediately after the TAG meeting, each proposed modification will be returned to the SDO with one of the following recommendations:

1.) Recommended for adoption as presented.

2.) Recommended for adoption with resolution of specific changes.

3.) Not recommended for adoption.

F. Revision Recommendations to SSDE

Within two weeks after the TAG meeting, the SDO Staff will resolve each recommended modification and the assembled modifications will be provided to the SSDE for final approval.

G. Revision Adoption and Implementation

Once approved by the SSDE, the ***SDG*** revision modifications will be assigned to the Design Technology Group Leader for final editing.

H. Official Distribution of Revisions as New *SDG* Version

Unless agreed otherwise, the ***SDG*** version modifications will be distributed within 4 weeks after receipt of the approved modifications from the SSDE. This time frame allows the Organization and Procedures Office to update the Standard Operating System and any electronic media prior to electronic distribution of the ***SDG.***

I.12 Training

No specific training is necessary for the use of this manual. Major revisions are often presented and discussed at the Biennial Structures Design Conference.

I.13 Forms

This manual requires no forms.

CHAPTER 7
CONCRETE STRUCTURES

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7.1 General

This Chapter contains information related to the design, reinforcing, detailing, and construction of concrete components. It also contains deviations from ***LRFD*** that are required in such areas as deck slab reinforcing and construction, post-tensioning design and detailing, and the design and use of adhesive anchors.

This Chapter covers the design of prestressed, pretensioned concrete components such as strand transfer and development and shear design.

When using Florida limerock coarse aggregate, use 90% of the value calculated for the Modulus of Elasticity in LRFD 5.4. For Florida limerock W_c is typically taken as 0.145 KCF.

7.2 Concrete Deck Slabs [5.13.1] [9.7]

7.2.1 Bridge Length Definitions for Deck Thickness and Finishing Req's.

For the purpose of establishing profilograph and deck thickness requirements, bridge structures are defined either as Short Bridges or Long Bridges. The determining length is the length of the bridge structure measured along the Profile Grade Line (PGL) of the structure. This includes the lengths of exposed concrete riding surface of the approach slabs. Based upon this established length, the following definitions shall apply:

A. Short Bridges: Bridge structures less than or equal to 300 feet in PGL length.

B. Long Bridges: Bridge structures more than 300 feet in PGL length.

7.2.2 Deck Thickness Determination

For new construction of “Long Bridges” other than inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8-½ inches. The 8-½ inch deck thickness includes a ½ inch additional, sacrificial thickness to be included in the dead load of the deck slab but omitted from its section properties.

For new construction of “Short Bridges” other than inverted-T Beam bridge superstructures, the minimum thickness of bridge decks cast-in-place (CIP) on beams or girders is 8 inches minimum.

The cast-in-place bridge deck thickness for Inverted-T Beam bridge superstructures with Inverted-T Beams spaced on 2’-0” centers is 6 ½ inches and 6 inches for bridges meeting the definition of Long and Short Bridges, respectively.

For “Major Widenings,” (see criteria in Chapter 9) the thickness of CIP bridge decks on beams or girders is 8 inches. However, whenever a Major Widening is selected by the Department to meet profilograph requirements, a minimum deck thickness of 8 ½ inches to meet the requirements and design methodology for new construction of the preceding paragraph, shall be used.

The thickness of CIP bridge decks on beams or girders for minor widenings will be determined on an individual basis but generally will match the thickness of the adjoining existing deck.

The thickness of all other CIP or precast concrete bridge decks is based upon the reinforcing cover requirements of Chapter 2, Table 2.2. For stay-in-place form restrictions, see Chapter 2.

Establish bearing elevations by deducting the determined thickness before milling, from the Finish Grade Elevations required by the Contract Drawings.

7.2.3 Grooving Bridge Decks

For new construction utilizing C-I-P bridge deck (floors) that will not be surfaced with asphaltic concrete Include the following item in the Summary of Pay Items:

Item No. 400-7 - Bridge Floor Grooving ....Sq. Yards

Quantity Determination: Determine the quantity of bridge floor grooving in accordance with the provisions of Article 400-22.4 of the “Specifications.”

7.2.4 Deck Slab Design [9.7.2][9.7.3]

A. Empirical Design Method

For Category 1 structures meeting the criteria in ***LRFD*** **[9.7.2.4]**, design deck slabs by the Empirical Design method of ***LRFD*** **[9.7.2].**

In lieu of the reinforcing requirements of ***LRFD*** **[9.7.2.5],** No. 5 bars at 12” centers shall be used in both directions in both the top and bottom layers. Two additional No. 5 bars shall be placed between the primary transverse top slab bars (4” nominal spacing) in the slab overhangs to meet the TL-4 loading requirements for the FDOT standard barriers. One of the additional bars shall be extended to the mid-point between the exterior beam and the first interior beam; the second additional bars shall be extended 3 feet beyond this mid-point. The maximum deck overhang shall not exceed 6 feet.

B. Traditional Design Method

For all Category 2 Structures and for Category 1 Structures that do not meet the requirements of ***LRFD* [9.7.2.4],** design deck slabs in accordance with the Traditional Design method of ***LRFD* [9.7.3]**.

For the deck overhang design and median barriers, the following minimum transverse top slab reinforcing may be provided (without further analysis) where the indicated minimum slab depths are provided and the total deck overhang is 6 feet or less. However, for 8" thick decks with 8'-0" soundwall traffic railings the deck overhang is limited to 1'-6". The extra slab depth for deck grinding is not included.

Traffic Railing Barrier (Test Level)	Slab Depth	A_s/ft (in^2)
32" F-Shape (TL-4)	8"	0.80
32" Vertical Face (TL-4)	8"	0.80
32" F-Shape Median (TL-4)	8"	0.40 **
8'-0" Soundwall (TL-4)	8"	0.93 ***
8'-0" Soundwall (TL-4)	10"	0.66 ***
42" F-Shape (TL-5)	10"	0.75
42" Vertical Face (TL-5)	8" (with 6" sidewalk)	0.40 *(0.40)

* Minimum reinforcing based on the 42" vertical face traffic railing mounted on a 6" thick sidewalk above an 8" deck with 2" cover to the top reinforcing in both the deck and sidewalk. Specify No. 4 Bars at 6" spacing placed tranversely in the top of the raised sidewalk.

** Minimum reinforcing required in both top and bottom of slab. Less reinforcing may be provided in the bottom, provided the sum of the top and bottom reinforcing is not less than 0.80 in^2/ft.

*** For the 8'-0" Soundwall, the area of top slab reinforcing 6 feet each side of deck expansion joints shall be increased by 30% to provide a minimum 1.21 in^2/ft for an 8" thick slab and 0.86 in^2/ft for a 10" thick slab. The development length of this additional reinforcing shall be evaluated and hooked ends provided for all bars when necessary.

For traffic railings located inside the exterior beam (other than median barriers), the minimum transverse reinforcing in the top of the slab may be reduced by 40% provided the bottom reinforcing is not less than the top reinforcing.

If the above reinforcing is less than or equal to twice the nominal slab reinforcing, then the extra reinforcing shall be cut-off 12 inches beyond the midpoint between the two exterior beams. If the above reinforcing is greater than twice the nominal slab reinforcing, then half of the extra reinforcing or up to 1/3 the total reinforcing shall be cut-off midway between the two exterior beams. The remaining extra reinforcing shall be cut-off at 3/4 of the two exterior beam spacing, but not closer than 2 feet from the first cut-off.

7.2.5 Traffic Railing Design Requirements

In lieu of the Traditional Design Method shown above, the following design values may be used to design the top transverse slab reinforcing, for the types listed:

Traffic Railing Type (Test Level)	M_c	T_u	L_d
32” F-Shape (TL-4)	15.7	7.1	7.67
32" Vertical Face (TL-4)	16.9	7.1	7.67
32" Median (TL-4)	15.3	3.5	7.67
8'-0" Soundwall (TL-4)	20.1 ***	***5.9	21.00
42" F-Shape (TL-5)	20.6	9.0	13.75
42” Vertical Face (TL-5)	25.8	10.6	12.50

*** For the 8’-0” Soundwall, increase the ultimate slab moment and tensile force by 30% for a distance of 6 feet each side of all deck expansion joints, except on approach slabs.

M_c (kip-ft/ft) - Ultimate slab moment from traffic railing impact. (The ultimate traffic railing and slab dead load moment at the traffic railing face (gutter line) must be added to M_c.)

T_u (kip/ft) - Ultimate tensile force to be resisted.

L_d (ft) - Distribution length along the base of the traffic railing at the gutter line near a traffic railing open joint (L_c + traffic railing height).

The following relationship must be satisfied:

$$\frac{T_u}{\varphi P_n} + \frac{M_u}{\varphi M_n} \leq 1.0$$

Where $\varphi = 1.0$ and $P_n = A_s f_y$ (tension steel only)

For locations inside the gutter line, these forces may be distributed over a longer length of L_d + 2D(tan 45º) feet. Where "D" equals the distance from the gutter line to the critical slab section. At open transverse deck joints, only half of the increased distribution length D(tan 45º) shall be used.

For flat slab bridges the transverse moment due to the traffic railing dead load may be neglected. The area of transverse top slab reinforcing determined by analysis, for flat slab bridges with edge traffic railings shall not be less than 0.30 in^2/ft within 4 feet of the gutter line for any TL-4 traffic railing or 0.40 in^2/ft within 10 feet of the gutter line for any TL-5 railing.

When more than 50% of the total transverse reinforcing must be cut off, a minimum of 2 feet shall separate the cut-off locations.

For traffic railings located inside the exterior beam, or greater than 5'-0" from the edge of flat bridges, the designer may assume that only 60% of the ultimate slab moment and tensile force are transferred to the deck slab on either side of the traffic railing.

7.2.6 Reinforcing Steel over Intermediate Piers or Bents

When CIP slabs are made composite with simple span concrete beams, and are cast continuous over intermediate piers or bents, design supplemental longitudinal reinforcing in the tops of slabs. Sized, space, and place reinforcing in accordance with the following criteria:

A. No. 5 Bars placed between the continuous, longitudinal reinforcing bars.

B. A minimum of 35 feet in length or 2/3 span whichever is less.

C. Placed symmetrically about the centerline of the pier or bent, with alternating bars staggered 5 feet.

7.2.7 Structures Continuous for Live Load

In structures designed continuous for live load, design supplemental longitudinal reinforcing extended beyond the point where the slab is in tension under any load combination.

7.2.8 Concrete Decks on Continuous Steel Girders

For longitudinal reinforcing steel within the negative moment regions of continuous, composite steel girder superstructures, comply with the requirements of ***LRFD* [6.10.3.7]**.

In addition, design the remainder of the deck with No. 5 longitudinal steel at 12 inch spacing in the top of the slab and No. 4 Bars at 12 inch spacing in the bottom of the slab.

7.2.9 Skewed Decks [9.7.1.3]

A. Reinforcing Placement when the Slab Skew is 15 Degrees or less:

Place the transverse reinforcement parallel to the skew for the entire length of the slab.

B. Reinforcing Placement when the Slab Skew is more than 15 Degrees:

Place the required transverse reinforcement perpendicular to the centerline of span. In this case, because the typical required transverse reinforcement cannot be placed full-width in the triangular shaped portions of the ends of the slab, the required amount of longitudinal reinforcing shall be doubled for a distance along the span equal to the beam spacing. In addition, three No. 5 Bars at 6” spacing, full-width, shall be placed parallel to the end skew in the top of each end of the slab.

Note: Regardless of the angle of skew, the traffic railing reinforcement cast into the slab need not be skewed.

7.2.10 Temperature and Shrinkage Reinforcement

For all cast in place decks, design temperature and shrinkage reinforcement per **LRFD [5.10.8]** except do not exceed 12” spacing and the minimum bar size is No 4.

7.3 Reinforcing Steel [5.4.3]

Steel reinforcing for concrete design shall be ASTM A615, Grade 60.

7.4 Prestressed, Pretensioned Components

7.4.1 Prestressed Piling [5.13.4.3]

For prestressed piling not subjected to significant flexure under service or impact loading, strand development shall be in accordance with ***LRFD*** **[5.11.4]** and **[5.8.2.3]**. Bending that produces cracking in the pile, such as that resulting from ship impact loading, is considered significant. A pile embedment of 4 feet into a footing is considered adequate to develop the strength of the pile.

For the standard square FDOT prestressed concrete piles (12 through 30 inch), a pile embedment of 4 feet into a reinforced concrete footing is considered adequate to develop the bending strength of the pile. The pile must be solid, or the pile void filled with structural concrete, within the 4-foot embedment length. A 1 foot embedment shall be considered a pinned head condition.

For the pinned pile head condition the strand development shall be in accordance with **LRFD [5.11.4] and [5.8.2.3]**.

The bending capacity versus pile cap embedment length relationship for prestressed piles with widths or diameters larger than 30 inches shall be established on a case-by-case basis.

Commentary:
The FDOT Structures Research Center conducted full scale testing of two 30 inch square concrete piles reinforced with prestressing steel and an embedded steel pipe. The piles, which were embedded 4 feet into a reinforced pile cap, developed the calculated theoretical bending strength of the section without strand slip. See FDOT Report No 98-9 "Testing of Pile to Pile Cap Moment Connection for 30" Prestressed Concrete Pipe-Pile". It was concluded that the confinement effects of the pile cap serve to improve the bond characteristics of the strand.

7.4.2 Prestressed Beams

The use of ASTM A416, Grade 270, low-relaxation, straight, prestressing strands is preferred for the design of prestressed beams. However, the requirements stipulated hereinafter apply to simply supported, fully pretensioned beams, whether of straight or depressed (draped) strand profile, except where specifically noted otherwise.

A. Bridges that contain varying span lengths, skew angles, beam spacing, beam loads, or other design criteria may result in very similar individual designs. The designer should consider the individual beams designs as a first trial subject to modifications by combining similar designs into groups of common materials and stranding based upon the following priorities:

   1.) 28-Day Compressive Concrete Strength (f'_c)
   2.) Stranding (size, number, and location)
   3.) Compressive Concrete Strength at Release (f'_{ci})
   4.) Shielding (Debonding)*

*Note: Full length shielding of strands in some beams to facilitate casting bed utilization of beams with slightly different strand patterns is prohibited.

Commentary:
Grouping beam designs in accordance with the priority list maximizes casting bed usage and minimizes variations in materials and stranding.

B. In analyzing stresses and/or determining the required length of debonding, stresses shall be limited to the following values:

   1.) Tension at top of beam at release (straight strand only):

   Outer 15 percent of design span: $f_t = 12\sqrt{f'_{ci}}$ (psi)

   Center 70 percent of design span: $f_t = 6\sqrt{f'_{ci}}$ (psi)

   2.) Tension at top of beam at release (depressed strands only): $f_t = 6\sqrt{f'_{ci}}$ (psi)

C. In order to achieve uniformity and consistency in designing strand patterns, the following parameters shall apply:

1.) Strand patterns utilizing an odd number of strands per row (a strand located on the centerline of beam) and a minimum side cover (centerline of strand to face of concrete) of 3 inches are required for all AASHTO and Florida Bulb-Tee beam sections except AASHTO Type V and VI beams for which a strand pattern with an even number of strands per row shall be utilized.

2.) Use of "L-shaped" longitudinal bars in the webs and flanges in end zone areas.

3.) The minimum compressive concrete strength at release shall be 4.0 ksi or 0.6 f'_c, whichever is the greater. Higher release strengths may be used and specified when required by the designer but generally should not exceed 0.8 f'_c.

4.) Prestressed beams shall be designed and specified to conform to classes and related strengths of concrete as shown in Table 7.1.

5.) The use of time-dependent, inelastic creep and shrinkage is not allowed in the design of simple span, pretensioned components either during design or construction. Prestress loss must be calculated in accordance with ***LRFD* [5.9.5]**.

Commentary:
The FDOT cannot practically control, nor require the Contractor to control, the construction sequence and materials for simple span precast, prestressed beams. To benefit from the use of refined time-dependent analysis, literally every prestressed beam design would have to be re-analyzed using the proper construction times, temperature, humidity, material properties, etc. of both the beam and the yet-to-be-cast composite slab.

6.) Stress and camber calculations for the design of simple span, pretensioned components shall be based upon the use of transformed section properties.

7.) When wide-top beams such as bulb-tees and AASHTO Types V and VI beams are used in conjunction with stay-in-place metal forms, the designer shall evaluate the edges of flanges of those beams to safely and adequately support the self-weight of the forms, concrete, and construction load specified in Section 400 of the ***FDOT Standard Specifications for Road and Bridge Construction***.

8.) The design thickness of the composite slab shall be provided from the top of the stay-in-place metal form to the finished slab surface, and the superstructure concrete quantity shall not include the concrete required to fill the form flutes.

Table 7.1 Concrete Classes and Strengths

Class of Concrete	28-Day Compressive Strength (f'_c) KSI
Class III*	5.0
Class IV	5.5
Class V (special)	6.0
Class V	6.5
Class VI	8.5

*Class III concrete may be used only when the superstructure environment is classified as Slightly Aggressive in accordance with the criteria in Chapter 2.

7.5 Precast Prestressed Slab Units [5.14.4.3]

To control the maximum camber expected in the field for precast prestressed slab units built without a topping, the design camber shall not exceed ¼ inch. For precast prestressed slab units built with a topping, the design camber shall not exceed 1 inch. Unless otherwise specified on the plans, the design camber shall be computed for slab concrete with an age of 120 days. The design camber shown on the plans shall be the value of camber due to prestressing minus the dead load deflection of the slab unit after all prestress losses.

In order to accommodate the enhanced post-tensioning system requirement of three levels of protection for strand, transverse post-tensioned pre-stressed slab units shall incorporate a double duct system. The outer duct shall be cast into the slab and sized to accommodate a differential camber of 1". The inner duct shall be continuous across all joints and shall be sized based upon the number of strands or the diameter of the bar coupler. Both the inner duct and the annulus between the ducts shall be grouted.

7.6 Florida Bulb-Tee Beams [5.14.1.2.2]

The minimum web thicknesses for Florida Bulb-Tee beams are:

A. Pretensioned Beams .... 6 ½ inches

B. Post-Tensioned Beams .... See Table 7.5

7.7 Precast Prestressed Double-Tee Beams

All bridge structures utilizing precast, prestressed double-tee beams shall conform with the design criteria and details provided in the ***Standard Drawings***.

7.8 Stay-in-Place (SIP) Slab Forms

For prestressed beam and steel girder superstructures, design and detail stay-in-place metal forms except where restricted for use by Chapter 2. State in the General Notes whether or not stay-in-place forms are permitted on the bridge.

7.9 Prestressed Beam Camber/Build-Up Over Beams

Unless otherwise required as a design parameter, beam camber for computing the build-up shown on the plans shall be based on an age of beam concrete of 120 days. In all cases, the age of beam concrete used for camber calculations shall be shown on the build-up detail as well as the value of camber due to prestressing minus the dead load deflection of the beam.

Commentary:
In the past, the FDOT has experienced significant slab construction problems associated with excessive prestressed, pretensioned beam camber. The use of straight strand beam designs, higher strength materials permitting longer spans, stage construction, long storage periods, improperly placed dunnage, and construction delays are some of the factors that have contributed to camber growth. Actual camber at the time of casting the slab equal to 2 to 3 times the initial camber at release is not uncommon.

7.10 General Criteria For Post-tensioned Bridges [5.14.2]

The design criteria included in this Chapter apply to the design of precast or cast-in-place post-tensioned concrete bridges in Florida. These criteria are to be used by any authority, consultant, or contractor engaged in design of post-tensioned structures for the FDOT. The criteria below applies to both concrete boxes and post-tensioned I-girders unless otherwise noted.

Prepare Contract drawings in accordance with ***LRFD*** Specifications. Specific requirements regarding post-tensioning design and details are described in this Chapter. Design concrete box girders to meet the requirements of Chapter 1 for maintenance, access, ventilation, and drainage.

7.10.1 Thermal Effects For Box Girder Structures [5.14.2.3.5]

A. For detailing purposes, take the normal mean temperature from Table 6.2.

B. For Seasonal Variation (expansion/contraction), refer to Table 6.2 for temperature ranges.

C. Show temperature setting variations for bearing and expansion joints on the bridge plans.

7.10.2 Creep and Shrinkage For Box Girder Structures [5.14.2.3.6]

Calculate creep and shrinkage strains and effects in accordance with ***LRFD*** using Relative Humidity of 75%.

7.10.3 Creep and Shrinkage For Post-tensioned I-Girder Structures

A. Calculate creep and shrinkage strains and effects in accordance with ***LRFD*** using Relative Humidity of 75%.

B. For the design of continuous prestressed concrete I-Girder superstructures, comply with the requirements of *LRFD* [5.4.2.3] by utilizing ACI 209 with the following design values:

1.) Ultimate Creep Coefficient 2.0

2.) Ultimate Shrinkage Strain.. 0.0004

3.) Beam Age when Deck is Cast .... 120 Days

C. These creep and shrinkage values include corrections for slump, humidity, and volume/surface ratio; and shall be used for both the beam and the deck slab.

Commentary:
A parametric study conducted by the FDOT's Structures Design Office indicates that the above values predict losses consistent with the AASHTO lump-sum loss approach. The correction factors applied to the basic creep and shrinkage values are average values. The values given above are subject to change as future research results become available.

7.10.4 Prestress

Design the structure for initial and final prestress forces.

A. Secondary Effects:

Prestress forces in continuous structures can result in substantial secondary effects. Also, in curved structures, draped web tendons will produce transverse bending stresses in the box cross-section due to the lateral component of forces arising from plan curvature. All such effects must be properly considered.

B. Deck Slab:

All box girder deck slabs shall be transversely post-tensioned. Where draped post-tensioning is used in deck slabs, consideration shall be given to the final location of the center of gravity of the prestressing steel within the duct. Critical eccentricities over the webs and at the centerline of box shall be reduced ¼ inch from theoretical to account for construction tolerances.

C. Tendon Geometry:

When coordinating design calculations with detail drawings, the EOR shall account for the fact that the center of gravity of the duct and the center of gravity of the prestressing steel are not necessarily coincidental.

D. Required Prestress:

Prestress forces are required to be shown on the drawings at anchorages at the stressing ends of tendons.

E. Internal/External Tendons:

External tendons shall remain external to the section without entering the top or bottom slab.

F. Strand Couplers:

Strand couplers as described in LRFD [5.4.5] are not allowed.

G. Post-tensioned Beams:

In designing pretensioned beams made continuous by field-applied post-tensioning, the pretensioning shall meet the following minimum criteria:

1.) The pretensioning acting alone shall comply with the minimum steel requirements of ***LRFD* [5.7.3.3.2]**.

2.) The pretensioning shall be capable of resisting all loads applied prior to post-tensioning, including a superimposed dead load equal to 50% of the uniform weight of the beam, without exceeding the stress limitation for pretensioned concrete construction.

3.) The initial mid span camber at release, including the effect of the dead load of the beam, is at least ½ inch. In computing the initial camber, the value of the modulus of elasticity of the concrete, E_c, shall be in accordance with Section 7.1 for the minimum required strength of concrete at release of the pretensioning force. The pretensioning force in the strands shall be reduced by losses due to elastic shortening and steel relaxation.

4.) Anchorage zones of post-tensioning tendons as well as beam segments in which ducts deviate both horizontally and vertically require integrated drawings showing all post-tensioning hardware and reinforcing steel.

5.) In designing pretensioned beams made continuous by field-applied post-tensioning, the pretensioning shall be designed such that the following minimum criteria are satisfied:

a.) The pretensioning acting alone shall comply with the minimum steel requirements of ***LRFD* [5.7.3.3.2]**.

b.) The pretensioning shall be capable of resisting all loads applied prior to post-tensioning, including a superimposed dead load equal to 50% of the uniform weight of the beam, without exceeding the stress limitation for pretensioned concrete construction.

7.10.5 Material

A. Concrete:

The minimum 28-day cylinder strengths of concrete shall be:

1.) Precast superstructure (including CIP joints) .... 5.5 ksi
2.) Precast pier stems .... 5.5 ksi
3.) Post-tensioned I-girders .... 5.5 ksi

B. Post-Tensioning Steel:

1.) Strand .... ASTM A416, Grade 270, low relaxation.
2.) Parallel wires .... ASTM A421, Grade 240.
3.) Bars .... ASTM A722, Grade 150.

C. Post-Tensioning Anchor set (to be verified during construction):

1.) Strand .... ½ inch
2.) Parallel wires .... ½ inch
3.) Bars 0

7.10.6 Expansion Joint Criteria

A. Design:

Design expansion joints for the full range of movement anticipated due to creep, shrinkage, elastic shortening, and temperature effects. The provisions of Chapter 6 shall also apply. Do not desing superstructures utilizing expansion joints within the span (i.e. ¼ point hinges).

B. Settings:

The setting of expansion joint recesses and expansion joint devices, including any precompression, shall be clearly stated on the drawings. Expansion joints shall be sized and set at time of construction for the following conditions:

1.) Allowance for opening movements based on the total anticipated movement resulting from the combined effects of creep, shrinkage, and temperature rise and fall. For box girder structures, compute creep and shrinkage from the time the expansion joints are installed through day 4000. Base temperature rise and fall on 120% of the maximum value given in Figure 8-1. Place a note on the plans stating that all expansion joints be installed after superstructure segment erection and deck profiling is completed for the entire bridge.

2.) To account for the larger amount of opening movement, expansion devices should be set precompressed to the maximum extent possible. In calculations allow for an assumed setting temperature of 85 degrees F. Provide a table on the plans giving precompression settings according to the prevailing conditions. Size Expansion devices and set to remain in compression through the full range of design temperature from their initial installation until a time of 4,000 days.

3.) A table of setting adjustments shall be provided to account for temperature variation at the time of installation. The table shall indicate the ambient air temperature at time of installation, and adjustments shall be calculated for the difference between the ambient air temperature and the mean temperature given in Figure 6-1.

C. Armoring:

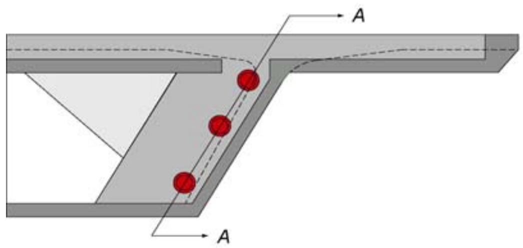
Concrete corners under expansion joint devices shall be provided with adequate steel armoring to prevent spalling or other damage under traffic. It is recommended that the armor be at least 4x4x5/8 galvanized angles anchored to the concrete with welded studs or similar devices. Horizontal concrete surfaces supporting the expansion joint device and running flush with the armoring shall have a finish acceptable for the device. All armoring shall have adequate vent holes to assure proper filling and compaction of the concrete under the armor.

D. Details For Post-Tensioned Bridges:

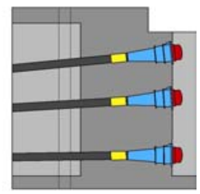
At expansion joints, provide a recess and continuous expansion joint device seat to receive the assembly, anchor bolts, and frames of the expansion joint, i.e. a finger or modular type joint. In the past, block-outs have been made in such seats to provide access for stressing jacks to the upper longitudinal tendon anchors set as high as possible in the anchor block. Lower the upper tendon anchors and re-arrange the anchor layout as necessary to provide access for the stressing jacks.

At all expansion joints, protect anchors from dripping water by means of skirts, baffles, v-grooves, or drip flanges. Drip flanges shall be of adequate size and shape to maintain structural integrity during form removal and erection.

Previous Practice

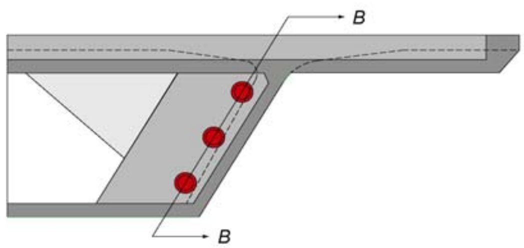


EJ Pier Segment End View

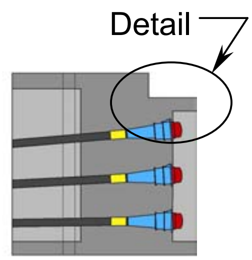


Section A-A

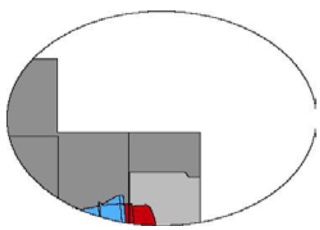
New Practice



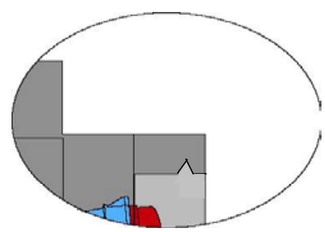
EJ Pier Segment End View



Section B-B



Detail A – Drip



Detail A – “V”

7.10.7 Erection Schedule and Construction System

A typical erection schedule and anticipated construction system shall be incorporated into the design documents in an outlined, schematic form. The assumed erection loads, along with times of application and removal of each of the erection loads, shall be clearly stated in the plans.

7.10.8 Const. Data Elev. and Camber Curve For Box Girder Structures

A. General:

Construction Data Elevations shall be based on the vertical and horizontal highway geometry; whereas, the Camber Curve shall be calculated and based on the assumed erection loads used in the design and the assumed construction sequence.

B. Construction Data Elevations:

This information shall be based on the highway geometry and shall be given in 3D space with “x”, “y”, and “z” coordinates. The data points shall be located at the centerline of the box and over each web of the box(es).

C. Camber Curve:

Camber Curve data shall be provided at the centerline of the box. Camber curve data is the opposite of deflections. Camber is the amount by which the concrete profile at the time of casting must differ from the theoretical geometric profile grade (generally a straight line) in order to compensate for all structural dead load, post-tensioning, long and short term time dependent deformations (creep and shrinkage), and effects of construction loads and sequence of erection.

7.10.9 Final Computer Run For Box Girder Structures

It is required that the final design shall be proven by a full longitudinal analysis taking into account the assumed construction process and final long term service condition, including all time related effects. The analysis shall be made using a computer program approved by SDO.

7.10.10 Integrated Drawings

Congested areas of post-tensioned concrete structures shall be shown on integrated drawings with an assumed post-tensioning system. Such areas include, but are not necessarily limited to, anchorages zones, areas containing embedded items for the assumed post-tensioning system, areas where post-tensioning ducts deviate both in the vertical and transverse directions, and other highly congested areas as determined by the engineer and/or the Department.

For curved structures, evaluate and accomodate possible conflicts between webs and external tendons. Also, check for conflicts between future post-tensioning tendons and permanent tendons.

The assumed post-tensioning system, embedded items, etc. shall be selected in a manner that will accommodate competitive systems using standard anchorage sizes of 4-0.6” dia, 7-0.6” dia, 12-0.6” dia, 19-0.6 dia and 27-0.6” dia. Integrated drawings utilizing the assumed system shall be defined to a scale and quality required to show double-line reinforcing and post-tensioning steel in two-dimension (2-D) and, when necessary, in complete three-dimension (3-D) drawings and details.

7.10.11 Post-tensioning Anchorages in Box Girders

Post-tensioning anchorages that are required in either the top or bottom slab of box girder bridges, whether for permanent or temporary post-tensioning, shall be anchored by means of interior “blisters,” face anchors or other approved means. Blockouts that extend either to the interior or exterior surfaces of the slabs are not permitted.

7.10.12 Epoxy Joining of Segments

All joints between precast segmental bridge segments shall contain epoxy on both faces. This requirement applies to substructure and superstructure precast units. Dry segment joints are not allowed.

7.11 Post-Tensioning Design [5.4.6][5.10.3.3]

7.11.1 Ducts and Tendons

Show offset dimensions to post-tensioning duct trajectories from fixed surfaces or clearly defined reference lines at intervals not exceeding 5 feet. When the radius of curvature of a duct exceeds one-half degree per foot, offsets shall be shown at intervals not exceeding 30 inches. In regions of tight reverse curvature of short sections of tendons, offsets shall be shown at sufficiently frequent intervals to clearly define the reverse curve.

Curved ducts that run parallel to each other or around a void or re-entrant corner shall be sufficiently encased in concrete and reinforced as necessary to avoid radial failure (pull-out into the other duct or void). In the case of approximately parallel ducts, the EOR shall consider the arrangement, installation, stressing sequence, and grouting in order to avoid potential problems with cross-grouting of ducts.

Detail post-tensioned bulb-tee beams to utilize round metal ducts only.

Table 7.2 Minimum Center-to-Center Duct Spacing:

Post Tensioned Bridge Type	*Minimum Center To Center Longitudinal Duct Spacing
Precast Segmental Balanced Cantilever Cast-In-Place Balanced Cantilever	8”, 2 times outer duct diameter, or outer duct diameter plus 4 ½ inches whichever is greater.
Spliced I-Girder Bridges	4 inches, outer duct diameter plus 1.5 times maximum aggregate size, or outer duct diameter plus 2 inches whichever is greater.
C.I.P. Voided Slab Bridges C.I.P. Multi-Cell Bridges	When all ducts are in a vertical plane – 4 inches, outer duct diameter plus 1.5 times maximum aggregate size, or outer duct diameter plus 2 inches whichever is greater. **For two or more ducts set side by side - outer duct diameter plus 3 inches.

* - Bundled tendons shall not be allowed.
** - The 3 inch measurement shall be measured in a horizontal plane.

Ducts for all post-tensioning bars shall be sized to be ½ inch larger than the diameter of the bar coupler.

Internal post-tensioning ducts are required to be positively sealed with duct couplers at all segment joints to eliminate the possibility of contamination. Therefore, internal tendons shall be designed and detailed to be perpendicular to all segment joints and all closure pours shall be a minimum of 18 inches.

To allow room for the installation of duct couplers, detail all external tendons to provide a 1 ½ inch clearance between the duct surface and the face of the concrete. Diablos are not allowed.

Detail both internal and external tendons to be placed in steel pipes when the radius is less than shown in the table below. Detail all deviation saddles to contain steel pipe regardless of the tendon radius.

Table 7.3 Minimum Tendon Radius

Tendon Size	Minimum Radius (ft)
19-0.5” dia, 12-0.6” dia	8’
31-0.5” dia, 19-0.6” dia	10’
55-0.5” dia, 37-0.6” dia	13’

All balanced cantilever bridges shall utilize a minimum of 4 positive moment external draped continuity tendons (2 per web) that extend to adjacent pier diaphragms.

Table 7.4 Min. Tendons Required for Critical Post-tensioned Sections:

Post Tensioned Bridge Element	Minimum Number of Tendons
Mid Span Closure Pour – C.I.P. and Precast Balanced Cantilever Bridges	Bottom slab – 2 per web Top slab – 1 per web (4-0.6 inch diameter minimum)
Span by Span Segmental Bridges	4 tendons per web
C.I.P. Multi-Cell Bridges	3 tendons per web
Spliced I-Girder Bridges	*3 per girder
Unit End Spans - C.I.P. and Precast Balanced Cantilever Bridges	3 tendons per web
Diaphragms - Transverse Post-Tensioning (When Vertical Post-Tensioning is Required)	6 if strength is provided by P.T. only 4 if strength is provided by combination of P.T. and mild reinforcing
Diaphragms – When Vertical Post-Tensioning is Required	4 **
Segments – (When Vertical Post-Tensioning is Required)	2 per web

* 3 girders minimum per bridge.
** 2 per additional cell

7.11.2 Minimum Dimensions for Post-tensioned Structural Elements

Table 7.5 Dimensions for sections containing post-tensioning tendons:

Post Tensioned Bridge Element	Minimum Thickness
Webs; I-Girder Bridges	8 inches or outer duct plus 5 inches whichever is greater.
Regions of Slabs without longitudinal tendons	8 inches, or as required to accommodate grinding, concrete covers, transverse and longitudinal P.T. ducts and top and bottom mild reinforcing mats, with allowance for construction tolerances whichever is greater.
Regions of slabs containing longitudinal internal tendons	9 inches, or as required to accommodate grinding, concrete covers, transverse and longitudinal P.T. ducts and top and bottom mild reinforcing mats, with allowance for construction tolerances whichever is greater.
Clear Distance Between Circular Voids - C.I.P. Voided Slab Bridges	Outer duct diameter plus 5 inches, or outer duct diameter plus vertical reinforcing plus concrete cover whichever is greater.
Segment Pier Diaphragms containing external post-tensioning	*6'-0"
Webs of C.I.P. boxes with internal tendons	For single column of ducts –12 inches, or 3 times outer duct diameter whichever is greater. **For two or more ducts set side by side – Web thickness shall be sufficient to accommodate concrete covers, longitudinal P.T. ducts, 3 inch min. spacing between ducts, vertical reinforcing, with allowance for construction tolerances.

*Pier segment halves with C.I.P. closure joint is acceptable.
** The 3 inch measurement shall be measured in a horizontal plane.

7.11.3 Box Girders

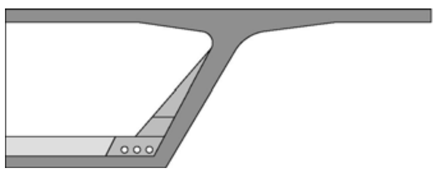
Provide continuous typical longitudinal mild reinforcing through all segment joints for Cast-in-place segmental construction.

All future post-tensioning shall utilize external tendons (bars or strands) and shall be detailed such that any one span can be strengthened independent of adjacent spans.

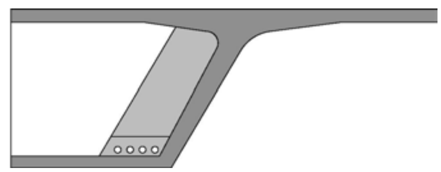
All anchor blisters for tendons shall terminate no closer than 12 inches to a joint between segments.

All interior blisters shall be set back from the joint a minimum of 12 inches. Provide a "V"-groove around the top slab blisters to isolate the anchorage from any free water.

Transverse bottom slab ribs are not allowed. Full height diaphragms directing the deviation forces directly into the web and slab shall be used in lieu of deviation saddles with transverse bottom slab ribs.

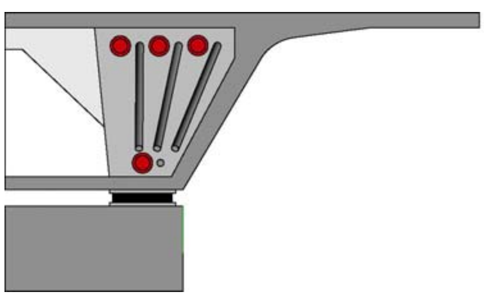


Previous Practice

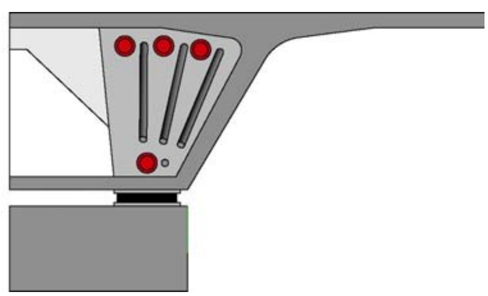


New Practice

Raised corner recesses in the top corner of pier segments at closure joints are not allowed. The typical cross section shall be continued to the face of the diaphragm. Tendon anchorages shall be located to make allowances for jack placement.



Previous Practice



New Practice

7.11.4 Post-Tensioned I-Girders

Individual deck block-outs for second stage post-tensioning access for I-Girder bridges shall not be allowed. Detail the deck in this region to be poured full width and extending one foot beyond the last anchorage upon completion of post-tensioning operations.

7.12 Adhesive Anchor Systems

7.12.1 General

Adhesive Anchor Systems are used to attach new construction to existing concrete structures. Adhesive Anchor Systems incorporate an adhesive bonding material and steel bar anchors installed in clean, dry holes drilled in hardened concrete. Anchors may be reinforcing bars or threaded rods depending upon the application. Do not use Adhesive Anchor Systems to splice with existing reinforcing bars in either non-prestressed or prestressed concrete applications unless special testing is performed and special, proven construction techniques are utilized.

For pre-approved Adhesive Anchor Systems, refer to the Department's **Qualified Products List (QPL)**, "Adhesive Bonding Material Systems for Structural Applications." Comply with Section 937 of the Specifications. Require that Adhesive Anchors be installed in accordance with manufacturer's recommendations for hole diameter and hole cleaning technique and meet the requirements of Section 416 of the Specifications.

Commentary:
Installation of Adhesive Anchor Systems in saturated, surface-dry holes; i.e., holes with damp surfaces but with no standing water, is not pre-approved or recommended by the Department. However, in the event such a condition is encountered during construction, the Department may consider approving continued installation, but only on an adjusted, case-by-case basis. The damp hole strength of products on the ***QPL*** *has been determined to be approximately 75% of the required dry hole strength.*

Unless special circumstances dictate otherwise, design Adhesive Anchor Systems for a ductile failure. A ductile failure requires embedment sufficient to ensure that failure will occur by yielding of the steel. In order to produce ductile failure, the following embedments may be assumed:

A. For Anchors in Tension:

That length of embedment which will develop 125 percent (125%) of the specified yield strength or 100 percent (100%) of the specified tensile strength, whichever is less.

B. For Anchors in Shear:

An embedment equal to seventy percent (70%) of the embedment length determined for anchors in tension.

In circumstances where ductile failure is not required, the design may be based upon the design strength of either the steel anchor or the adhesive bond, whichever is less.

Adhesive Anchor Systems meeting the specifications and design constraints of this article are permitted for horizontal, vertical, or downwardly inclined installations, only. Overhead or upwardly inclined installations of Adhesive Anchors are prohibited. Use QPL listed adhesive bonding to splice prestressed piling.

7.12.2 Notation

The following notation is used in this Article:

A_e = effective tensile stress area of steel anchor (may be taken as 75% of the gross area for threaded anchors). [in^2]

A_{n0} = $(16d)^2$, effective area of a single Adhesive Anchor in tension; used in calculating Ψ_{gn} (See Figure 7-1). [in^2]

A_n = effective area of a group of Adhesive Anchors in tension; used in calculating Ψ_{gn}, defined as the rectangular area bounded by a perimeter spaced 8d from the center of the anchors and limited by free edges of concrete (See Figure 7-1). [in^2]

A_{v0} = $4.5(c^2)$, effective breakout area of a single Adhesive Anchor in shear; used in calculating Ψ_{gv} (See Figure 7-2). [in^2]

A_v = effective area of a group of Adhesive Anchors in shear and/or loaded in shear where the member thickness, h, is less than 1.5c; used in calculating Ψ_{gv}, (See Figure 7-2). [in^2]

c = anchor edge distance from free edge to centerline of the anchor). [in]

d = nominal diameter of Adhesive Anchor. [in]

$f'c$ = minimum specified concrete strength. [ksi]

f_y = minimum specified yield strength of Adhesive Anchor steel. [ksi]

f_u = minimum specified ultimate strength of Adhesive Anchor steel. [ksi]

h = concrete member thickness. [in]

h_e = embedment depth of anchor. [in]

N_c = tensile design strength as controlled by bond for Adhesive Anchors. [kips]

N_n = nominal tensile strength of Adhesive Anchor. [kips]

N_o = nominal tensile strength as controlled by concrete embedment for a single Adhesive Anchor. [kips]

N_s = design strength as controlled by Adhesive Anchor steel. [kips]

N_u = factored tension load. [kips]

s = Adhesive Anchor spacing (measured from centerlines of anchors). [in] When using Type HSHV adhesives, the minimum anchor spacing is 12d, unless approved by the State Structures Engineer.

V_c = shear design strength as controlled by the concrete embedment for Adhesive Anchors. [kips]

V_s = design shear strength as controlled by Adhesive Anchor steel. [kips]

V_u = factored shear load. [kips]

T' = 1.08 ksi nominal bond strength for general use products on the QPL (Type V and Type HV). 1.83 ksi nominal bond strength for Type HSHV adhesive products on the QPL for traffic railing barrier retrofits only.

$\varnothing_c$ = 0.85, capacity reduction factor for adhesive anchor controlled by the concrete embedment, ($\varnothing_c$ =1.00 for extreme event load case).
$\varnothing_s$ = 0.90, capacity reduction factor for adhesive anchor controlled by anchor steel.
Ψ_e = modification factor, for strength in tension, to account for anchor edge distance less than 8*d* (1.0 when $c \geq 8d$).
Ψ_{gn} = strength reduction factor for Adhesive Anchor groups in tension (1.0 when $s \geq 16d$).
Ψ_{gv} = strength reduction factor for Adhesive Anchor groups in shear and single Adhesive Anchors in shear influenced by member thickness (1.0 when $s \geq 3.0c$ and $h \geq 1.5c$).

7.12.3 Design Requirements for Tensile Loading

Use Equation 7-2 to determine the design tensile strength for Adhesive Anchor steel:

$$N_s = \varnothing_s A_e f_y \qquad \textbf{[Eq. 7-2]}$$

Use Equation 7-3 to determine the design tensile strength for Adhesive Anchor bond:

$$N_c = \varnothing_c \Psi_e \Psi_{gn} N_o \qquad \textbf{[Eq. 7-3]}$$

where:

$$N_o = T'\pi d\, h_e \qquad \textbf{[Eq. 7-4]}$$

For anchors with a distance to a free edge of concrete less than 8*d*, but greater than or equal to 3*d*, a reduction factor, Ψ_e, as given by Equation 7-5 shall be used. For anchors located less than 3*d* from a free edge of concrete, an appropriate strength reduction factor shall be determined by special testing. For anchors with an edge distance greater than 8*d*, Ψ_e may be taken as 1.0.

$$\psi_e = 0.70 + 0.30\,(c/8d) \qquad \textbf{[Eq. 7-5]}$$

For anchors loaded in tension and spaced closer than 16d, a reduction factor, Ψ_{gn}, given by Equation 7-6 shall be used. For anchor spacing greater than 16d, Ψ_{gn} shall be taken as 1.0.

$$\Psi_{gn} = (A_n / A_{no}) \qquad \textbf{[Eq. 7-6]}$$

7.12.4 Design Requirements for Shear Loading

Adhesive Anchors loaded in shear shall be embedded a distance of not less than 6*d*.

For Adhesive Anchors loaded in shear, the design shear strength controlled by anchor steel is determined by Equation 7-7:

$$V_s = Ø_s\ 0.7\ A_s\ f_y \qquad \textbf{[Eq. 7-7]}$$

For Adhesive Anchors loaded in shear, the design shear strength controlled by concrete breakout for shear directed toward a free edge of concrete is determined by Equation 7-8:

$$V_c = Ø_c\ \Psi_{gv}\ 0.4534\ c^{1.5} \sqrt{f'_c} \qquad \textbf{[Eq. 7-8]}$$

For anchors spaced closer than 3.0c, a reduction factor, Ψ_{gv}, given by Equation 7-9 shall be used. For anchor spacing greater than 3.0c, Ψ_{gv} shall be taken as 1.0.

$$\Psi_{gv} = A_v / A_{v0} \qquad \textbf{[Eq. 7-9]}$$

7.12.5 Interaction of Tensile and Shear Loadings

The following linear interaction between tension and shear loadings given by Equation 7-10 shall be used unless special testing is performed:

$$(N_u / ØN_n) + (V_n / ØV_n) \leq 1.0 \qquad \textbf{[Eq. 7-10]}$$

In Equation 7-10, $Ø_n$ is the smaller of the design tensile strength controlled by the Adhesive Anchor steel (Equation 7-2) or the design tensile strength as controlled by Adhesive Anchor bond (Equation 7-3). $ØV_n$ is the smaller of the design shear strength controlled by the Adhesive Anchor steel (Equation 7-7) or the design shear strength as controlled by concrete breakout (Equation 7-8).

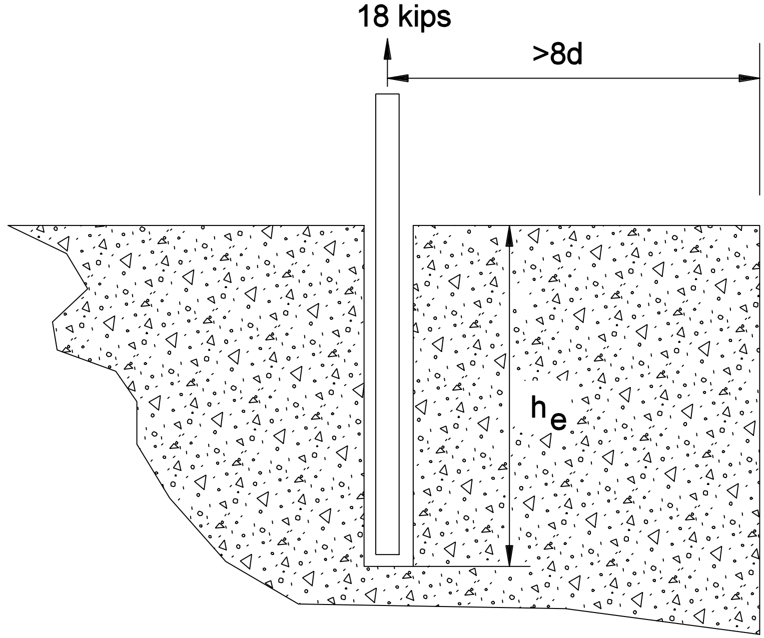
Commentary:
If Adhesive Anchor Systems are required to act as dowels from existing concrete components such that the existing reinforcing steel remains fully effective over its length, then the Adhesive Anchor System must be installed to a depth equal to the development length of the existing reinforcing steel. In this case, the required reinforcing steel spacing, covers, etc. apply to both the existing reinforcing steel and the Adhesive Anchor System. There is, however, no additional benefit to the Adhesive Anchor System to install anchors to a greater depth than required by this Article.

7.12.6 Example 1 - Single Anchor Away from Edges and Other Anchors

Design an adhesive anchor using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchor is located more than 8 anchor diameters from edges and is isolated from other anchors. The anchor embedment length is to be sufficient to ensure steel failure.

Given:

N_u	=	18.0 kips
f_y	=	100.0 ksi
f_u	=	125.0 ksi
T'	=	1.08 ksi



Section

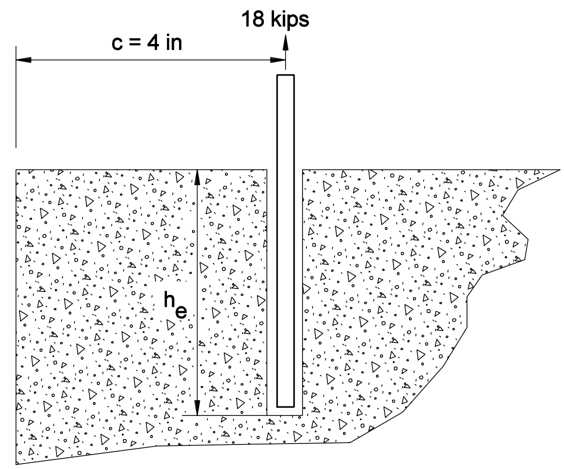
Design Procedure – Example 1	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = Ø_s A_e f_y$ where: $Ø_s = 0.9$ $A_e = 0.75 (\pi d^2/4)$ $f_y = 100$ ksi
Substituting and solving for *d*:	$18 = (0.9)[0.75 (\pi d^2/4)](100)$ d = 0.583” therefore, use 5/8” threaded rod
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no edge or spacing concerns, Ψ_e *and* Ψ_{gn} may be taken as unity.	$N_c = Ø_c \Psi_e \Psi_{gn} N_o$ (for embedment) where: $Ø_c = 0.85$ $\Psi_e, \Psi_{gn} = 1.0$ (no edge/spacing concern) $N_o = T' \pi d h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	$N_{c\,(req'd)} = 1.25 A_e f_y \leq A_e f_u$
Determine the effective area for a 5/8” threaded rod:	$A_e = 0.75 (\pi 0.625^2/4)$ $A_e = 0.23$ in^2
Determine the required tension force, $N_{c(req'd)}$, to ensure ductile behavior.	$N_{c(req'd)} = 1.25 A_e f_y \leq A_e f_u$ $N_{c(req'd)} = 1.25 (.23)(100) \leq (.23)(125)$ $N_{c(req'd)} = 28.75$ kips = 28.75 kips therefore, use $N_{c(req'd)} = 28.75$ kips
Substituting and solving for h_e:	$28.75 = 0.85 (1.0) (1.0) (1.08) \pi (.625) h_e$ $h_e = 16$ in

7.12.7 Example 2 - Single Anchor Away from Other Anchors but Near Edge

Design an adhesive anchor using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchor is located 4 inches from an edge but is isolated from other anchors. The anchor embedment length is to be sufficient to ensure steel failure.

Given:

N_u	=	18.0 kips
f_y	=	100.0 ksi
f_u	=	125.0 ksi
T'	=	1.08 ksi



Section

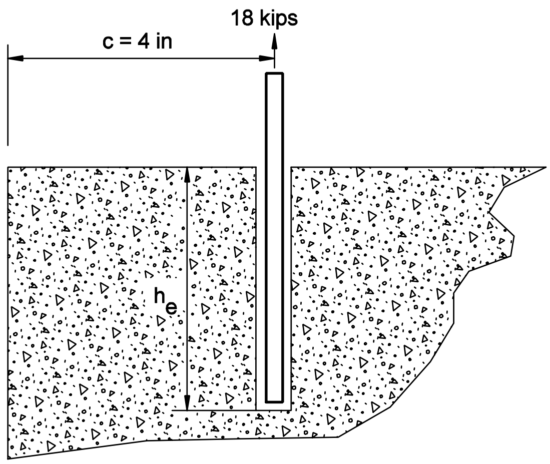
Design Procedure-Example 2	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = Ø_s A_e f_y$ where: $Ø_s$ = 0.9 A_e = $0.75 (\pi d^2/4)$ f_y = 100 ksi
Substituting and solving for *d*:	$18 = (0.9)[0.75 (\pi d^2/4)](100)$ d = 0.583 in therefore, use a 5/8" threaded rod
Step 2 - Determine required embedment length to ensure steel failure	
Basic equation for embedment length calculation. Since there are no spacing concerns, Ψ_{gn} may be taken as unity, and, since the edge distance (4 in) is less than 8*d* (5 in), the edge effect, Ψ_e , will need to be evaluated.	$N_c = Ø_c \Psi_e \Psi_{gn} N_o$ (for embedment) where: $Ø_c$ = 0.85 Ψ_{gn} = 1.0 (no spacing problem) $N_0 = T' \pi d h_e$
For ductile behavior it is necessary to embed the anchor sufficiently to develop 125% of the yield strength or 100% of the ultimate strength, whichever is less.	$N_{c(req'd)} = 1.25 A_e f_y \le A_e f_u$
Determine the effective area for a 5/8" threaded rod:	$A_e = 0.75 (\pi\, 0.625^2/4)$ $A_e = 0.23$ in^2
Determine the required tension force, $N_{c(req'd)}$, to ensure ductile behavior.	$N_{c(req'd)} = 1.25 A_e f_y \le A_e f_u$ $N_{c(req'd)} = 1.25 (.23)(100) \le (.23)(125)$ $N_{c(req'd)}$ = 28.75 kips = 28.75 kips Therefore, use $N_{c(req'd)}$ = 28.75 kips
Determine edge effect factor, Ψ_e. Note: $c_{cr} = 8d$	$\Psi_e = 0.70 + 0.30 (c/8d)$ $\Psi_e = 0.70 + 0.30 [4/(8)(.625)]$ $\Psi_e = 0.94$
Substituting and solving for h_e:	$28.75 = 0.85 (1.0)(0.94)(1.08)\, \pi\, (.625)\, h_e$ h_e = 16.98 inches

7.12.8 Example 3 - Two Anchors Spaced at 8 inches, 4 inches from Edge

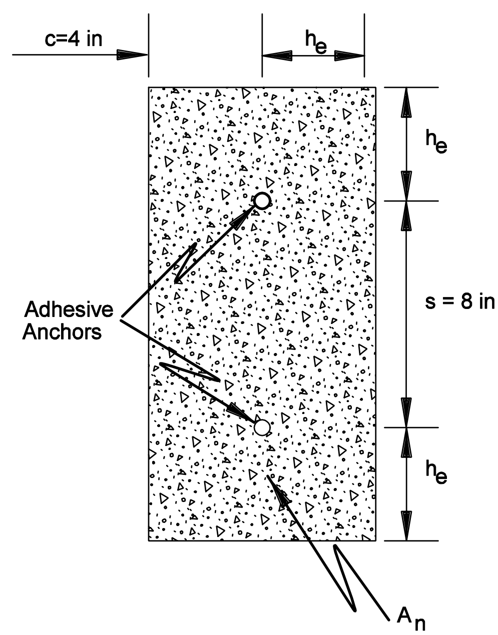
Design a group of two adhesive anchors using threaded rod (ASTM A193, Grade B7) for a factored tension load of 18 kips. The anchors are located 4 inches from an edge and are spaced 8 inches apart. Steel failure is not required.

Given:

N_u	=	18.0 kips
c	=	4 inches
s	=	8 inches
f_y	=	100 ksi
f_u	=	125 ksi
T'	=	1.08 ksi



Section



Plan

Design Procedure-Example 3	Calculation
Step 1 - Determine required rod diameter	
Determine the required diameter of the threaded rod by setting the factored tension load equal to the design steel strength.	$N_u = N_s$
The effective area for the threaded rod may be taken as 75% of the gross area. As with reinforcing bars, the minimum specified yield strength of the rod is used to determine the required diameter.	$N_s = Ø_s A_e f_y$ Where: $Ø_s = 0.9$ $A_e = (2)\ 0.75\ (\pi d^2/4)$ $f_y = 100$ ksi
Substituting and solving for *d*:	$18 = (0.9)(2)[0.75\ (\pi d^2/4)](100)$ $d = 0.412$ in Although a ½" threaded rod is OK, use a 5/8" threaded rod to minimize embedment length
Design steel strength	$N_s = 0.9(2)(0.75)(\pi 0.625^2/4)(100)$ $N_s = 41.4$ kips > 18 kips therefore; OK
Step 2 - Determine required embedment length	
Basic equation for embedment length calculation. Since there are edge or spacing concerns, Ψ_e *and* Ψ_{gn} will need to be determined.	$N_c = Ø_c\ \Psi_e\ \Psi_{gn}\ N_0$ (for embedment) where: $Ø_c = 0.85$ Ψ_e and Ψ_{gn} are calculated below $N_0 = T'\pi d\ h_e$
Determine edge effect factor, Ψ_e.	$\Psi_e = 0.70 + 0.30\ (c/8d)$ $\Psi_e = 0.70 + 0.30\ [4 / (8)(.625)]$ $\Psi_e = 0.94$
Determine group effect factor, Ψ_{gn}.	$\Psi_{gn} = A_n / A_0$ $\Psi_{gn} = (4 + 8d)[8 + 2(8d)]/(16)^2]$ $\Psi_{gn} = (4 + (8)(0.625)[8 + 2(8)(0.0625)]/[16(0.625)]^2$ $\Psi_{gn} = 1.62$
Substituting and solving for h_e.	$18 = (0.85)(1.62)(0.94)(1.08)\ \pi(.625)h_e$ h_e = 6.55 inches *say 7* inches *)* therefore OK
Design adhesive bond strength.	$N_c = (0.85)(1.62)(0.94)(1.08)\pi\ (.625)(7)$ $N_c = 19.21 > 18$. Therefore OK
Step 3 - Final Design Strength	
Strength as controlled by steel.	N_s = 41.4 kips > 18 kips. Therefore OK
Strength as controlled by adhesive bond.	N_c = 19.21 kips > 18 kips. Therefore OK
Final Design.	2 – 5/8" anchors embedded 7 in

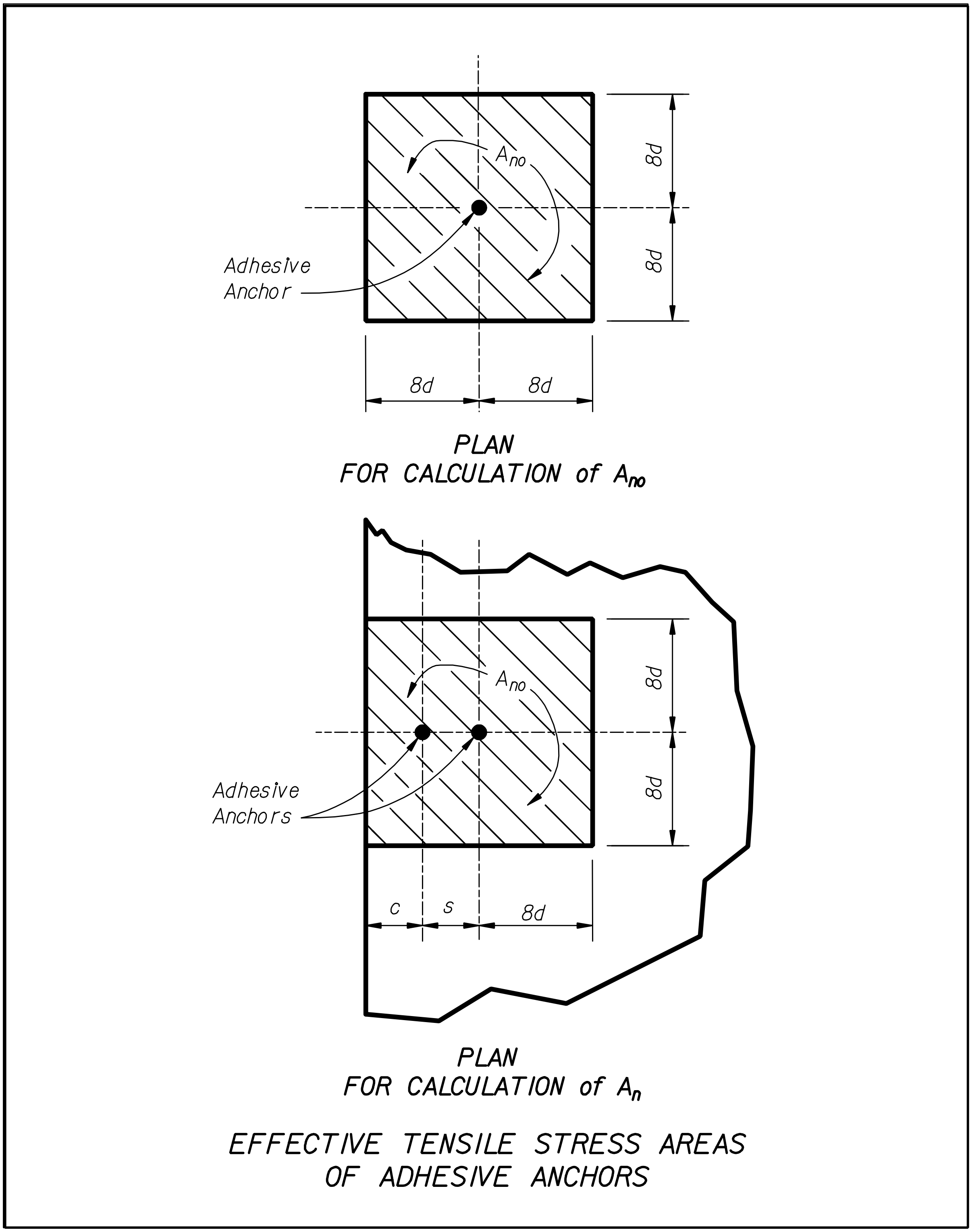

A_{no}
Adhesive
Anchor
8d
8d
8d
8d
PLAN
FOR CALCULATION of A_{no}
A_{no}
Adhesive
Anchors
8d
8d
c
s
8d
PLAN
FOR CALCULATION of A_n
EFFECTIVE TENSILE STRESS AREAS
OF ADHESIVE ANCHORS

Figure 7-1

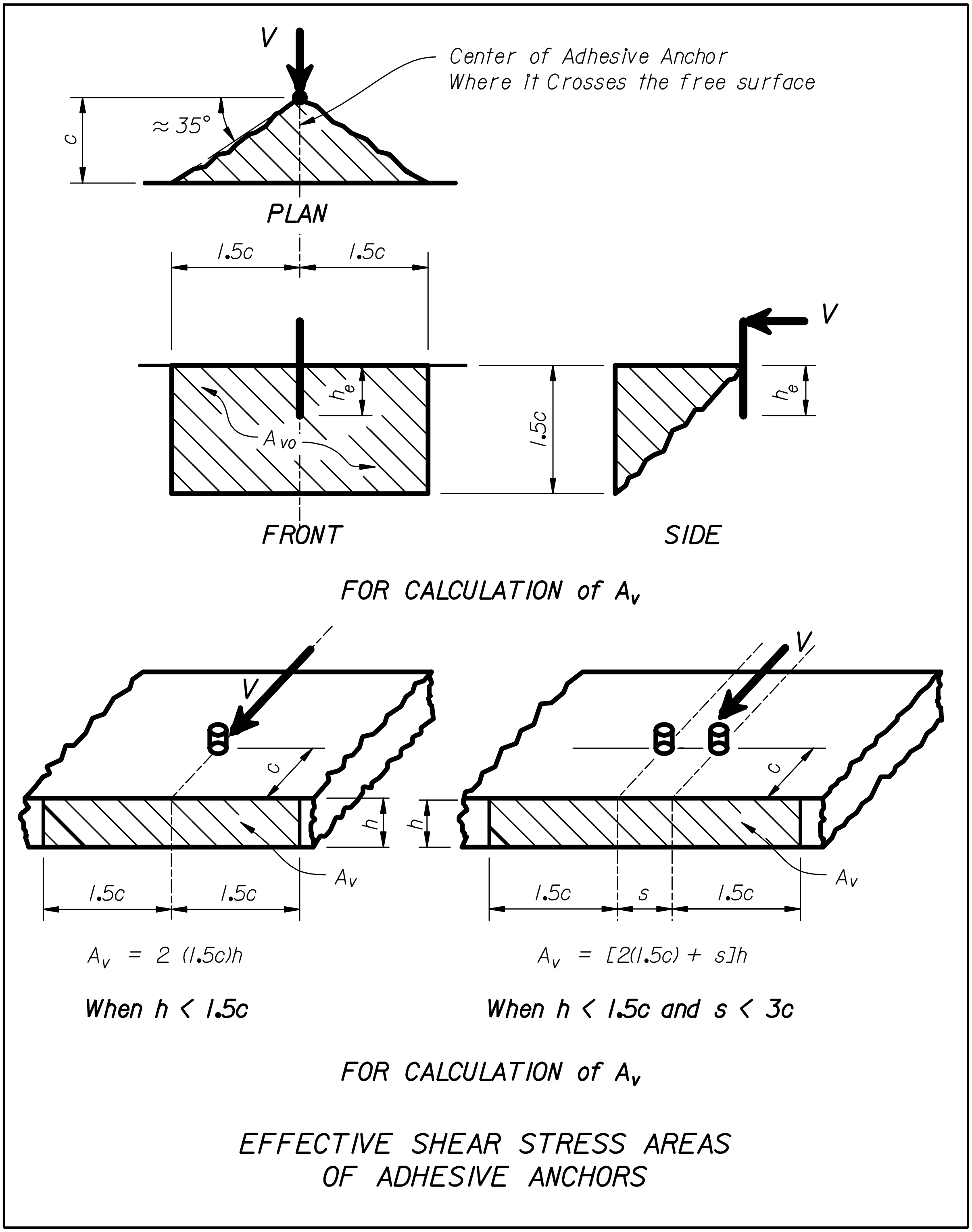

V
Center of Adhesive Anchor
Where It Crosses the free surface
c
≈ 35°
PLAN
1.5c
1.5c
V
h_e
A_{vo}
1.5c
h_e
FRONT
SIDE
FOR CALCULATION of A_v
V
V
c
c
h
h
A_v
A_v
1.5c
1.5c
1.5c
s
1.5c
A_v = 2 (1.5c)h
A_v = [2(1.5c) + s]h
When h < 1.5c
When h < 1.5c and s < 3c
FOR CALCULATION of A_v
EFFECTIVE SHEAR STRESS AREAS
OF ADHESIVE ANCHORS

Figure 7-2